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EVALUATION OF THE OXIDATION DITCH AS A MEANS OF WASTEWATER TREATMENT IN ONTARIO

THE ONTARIO WATER RESOURCES COMMISSION

EVALUATION OF THE OXIDATION DITCH
AS A MEANS OF
WASTEWATER TREATMENT IN ONTARIO

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SUMMARY

Several years ago the oxidation ditch was developed in the Netherlands, as a low cost means of secondary sewage treatment for small communities, which could ill afford the expensive conventional treatment systems. Since that time several hundred plants have been put into operation in Western Europe. More recently about a dozen oxidation ditch plants have been installed in Canada and the United States.

To determine whether the oxidation ditch would be suitable for use in Ontario, a preliminary review of pertinent literature and accumulated operating data was followed by an inspection tour of oxidation ditch installations in the State of Oregon, the Province of British Columbia and the Province of Saskatchewan. Discussion with government officials, consulting engineers and local authorities provided first-hand information regarding the performance, method and cost of construction, and operating and maintenance problems of the oxidation ditch. Operating records were analyzed. Tests were made to indicate the condition of the process.

It may be concluded on the basis of the acquired information, that the oxidation ditch treatment system is rather inexpensive to construct and simple to operate, and that it produces an acceptable effluent consistently. Therefore, the system merits consideration as an alternate means of waste treatment for small municipalities in Ontario. A comparative cost study conducted by the consulting engineer will indicate the most economical system for a particular locality, whether a lagoon, aerated lagoon or oxidation ditch.

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1. INTRODUCTION

After a preliminary acquaintance with the modified activated sludge process known as "The Oxidation Ditch", the interest of the OWRC in it began to grow, resulting in a review of available literature and an acquiring of operating data and auxiliary information preferably from North American installations. It became readily apparent, that additional information was urgently needed, before a decision could be made on the suitability and applicability of the oxidation ditch in Ontario.

Since the oxidation ditch was specifically developed for small communities, the treatment process should be competitive with sewage lagoons both with respect to capital and operating costs and required land area, to be of practical value in Ontario.

Therefore it was decided to conduct an inspection tour of several oxidation ditch installations in the State of Oregon, and Western Canada. The purpose of the tour was (a) to observe the operation of the plants and to evaluate the operating records where available, (b) to meet with State and Provincial Departments of Health to learn their opinion on the treatment process, (c) to discuss operating and maintenance problems with the local authorities in charge of the plants, and (d) to meet with several consulting engineers to learn their experience with respect to design and construction.

It should be realized that such an inspection tour will amount to a compromise effort between (a) full reliance on literature reports and operating data received from the respective authorities, and (b) the carrying out of a pilot, or field scale study on an oxidation ditch constructed in Ontario, and operating under local conditions.

2. PRINCIPLES OF OPERATION OF THE OXIDATION DITCH

The oxidation ditch treatment process was the result of an exhaustive long term research project conducted by the Research Institute for Public Health Engineering T.N.O., the Netherlands (1), where the urgent need was realized for a reliable and economical means of sewage treatment for small communities, since conventional treatment methods were too costly.

The oxidation ditch is essentially an extended aeration process providing long term aeration, in excess of 24 hours. Sufficient oxygen is provided to stabilize the primary solids, as well as to remove the dissolved and colloidal BOD of the wastewater. An acceptable, well-purified effluent is produced, along with a small excess of aerobically digested sludge. This means that the whole treatment plant needs to consist of an aeration tank and a means of sludge settlement. The excess sludge can be withdrawn periodically, and dried on drying beds, without causing obnoxious odours, thus primary sedimentation and sludge digestion become superfluous.

The basic element of the plant being the aeration tank, an attempt was successfully made to keep the construction of it as simple and cheap as possible. A shallow ditch in the shape of a racetrack circuit was built as the aeration tank. (see Fig. 1) Since the ditch was used with a shallow depth of only 3 feet, expensive concrete structures were unnecessary. It was found, that in stable soils an earthen ditch would suffice, the sides having a slope of 1:1 or 1:1.5. Grass sods were used originally to reinforce the sides of the ditch and to serve as protection against erosion.

A modification of the Kessener brush was developed for use as air supply. The aeration rotor, being horizontally mounted, provided also circulation of the liquid in the ditch keeping the mixed liquor solids in suspension. A so-called cage rotor, 70 cm (27½ in.) in diameter was used predominantly. (see Fig. 2).

Again in an attempt to keep construction costs to a minimum, the aeration ditch was used also as a sludge separation unit by means of intermittent operation of the rotor. A neat schedule was devised for timer operation of the rotor and the raw sewage feed pump. When the rotor was operating, the raw sewage flow was stored in the wet well and in the sewer system in the vicinity of the plant. The aerator was turned off after about 4½ hours aeration. The mixed liquor solids were allowed to settle for a period of one hour, then the raw sewage pump started up, and the flow of sewage entering the ditch on one end displaced the clarified effluent through the overflow siphon at the other end of the ditch. The operation of the pump was float controlled, therefore when the level in the wet well had been reduced to a minimum level, the pump stopped functioning, the discharge of effluent was stopped and the rotor was started up again to repeat the cycle. Float controls in the pump well were positioned so that 1/4 of the average dry weather flow was admitted per cycle.

It was recognized that the intermittent pump operation with the resulting storage of sewage would cause problems under certain conditions. The risk of shortcircuiting was also realized during the displacement of clarified effluent, with subsequent deterioration of effluent quality. Several designs were tried out to facilitate continuous separation of the sludge and thus continuous feeding of raw sewage and aeration. These designs consisted mostly of modifications of the basic ditch layout, without resorting to a separate clarifier with return sludge pump, again to keep the capital cost of the system to a minimum. In this connection it is interesting to note that most installations in North America are equipped with an outside clarifier. (see Fig. 3).

The experimental work (1) was conducted over a period of two years on an installation designed for a contributory population of 600. Subsequent measurements of raw sewage BOD and the daily flow showed a maximum loading of 365 population equivalents of 54 g BOD (= 0,12 lb.) or 256 population equivalents of 0.17 lb. The BOD loading was 12.2 lb./1,000 cu. ft. per day, or 4.9 lb. BOD/100 lb. MLSS based on a MLSS concentration of 4,000 mg/1. The observed average daily flow was 70 m^3 or 18,500 gallons (= 15,400 Imp. gallons), giving a retention time of 34 hours. The expected dry weather flow

was 40 m^3 per day. This compared with a measured D.W.F. of 38 m^3 per day or 10,000 gallons (=8,400 Imp. gallons). This flow occurred during the winter period when frost penetration eliminated groundwater infiltration.

Effluent BOD concentrations were generally 10 mg/l or less, thus the treatment system proved capable of an average removal efficiency of 97 per cent on the basis of an average BOD of 280 mg/l in the raw sewage. Precautions were taken not to include the oxygen demand of nitrifying organisms in the effluent BOD determinations. Experimental results also showed the development of a well-settleable sludge. Sludge volume index values were consistently low with a maximum of 74 ml/hr/g. The mixed liquor contained an average of 3,000 mg/l suspended solids, with a maximum concentration of 5,450 mg/l. The ash content varied between 20 and 32 per cent. This does not necessarily indicate a highly mineralized sludge.

Although no specific data were provided (1) regarding the oxygenation capacity of the rotor installed in the experimental plant, sampling showed adequate concentrations of dissolved oxygen in the mixed liquor. At the start of an aeration period the D.O. concentration varied between 0 and 1 mg/l, increasing to 3 and 4 mg/l towards the end of the 4 1/2 hour aeration period.

Recent experience with a full-scale oxidation ditch plant (2) showed some evidence of a dual process of nitrification and denitrification occurring in the mixed liquor. Under conditions of adequate aeration complete nitrification of the free ammonia caused a nitrate content of 50 mg/l N in the plant effluent. To prevent sludge rising in the clarifier due to denitrification the oxygenation capacity of the rotor was reduced by decreasing the rotor immersion and/or the hours of aeration. As a result the nitrate concentration in the effluent dropped to near zero. After maintaining the reduced rate of aeration for several months, chemical analysis of the effluent showed low concentrations of ammonia and nitrate, virtually no dissolved oxygen, and low BOD's (average of 14 samples 5 mg/l). The mixed liquor dissolved oxygen content was zero at a point 10 m (32.8 ft.) upstream of the rotor. It is very interesting to note that similar observations were made during the Inspection Tour, at least with respect to dissolved oxygen in mixed liquor and effluent, and to BOD of the effluent.

In this same context reference should be made to the work of Johnson and Schroepfer (3), on the removal of nitrogen by nitrification and denitrification in a modification of the activated sludge process. The modification consisted essentially of adding a holding tank, in which the aeration tank mixed liquor was kept in circulation in the absence of oxygen. The only source of oxygen was the chemically-bound oxygen of the nitrate content. Under these conditions the nitrate nitrogen was reduced to free nitrogen. The rate of nitrate removal could be significantly increased by adding untreated wastewater to the holding tank.

The design of the oxidation ditch combines an aeration tank and a holding tank into a single unit: the aeration rotor circulates the mixed liquor through the whole ditch, but aeration occurs only in the vicinity of the rotor. As a result the dissolved oxygen content of the mixed liquor will continuously decrease from a maximum value immediately downstream of the rotor to a minimum value upstream of the rotor. Depending on the oxygen requirements of the system and on the oxygenation capacity of the rotor, minimum D.O. values approach zero mg/l. Since raw wastewater is fed continuously to the ditch, it may serve to accelerate the denitrification process, when the mixed liquor passes through the "anaerobic" zone of the ditch.

Although dissolved oxygen levels of 0.5 mg/l or less are unacceptable for the conventional activated sludge process, it would appear that the biochemical processes in the oxidation ditch mixed liquor are not adverseely affected by low concentrations of dissolved oxygen. This may be the result of the very low loadings for which an oxidation ditch is designed, i.e. 0.05 lb. BOD per lb. MLSS per day.

Pasveer (2) is careful to point out that additional work is necessary, before a definite statement can be made with respect to denitrification in the oxidation ditch. He also indicates that a strongly nitrifying flora must be present before denitrification can start under conditions of reduced aeration. His findings show that denitrification may be possible without extra power costs.

After completion of the basic research on the oxidation ditch in 1957, the treatment process was quickly accepted. Several hundred oxidation ditches have been installed in the

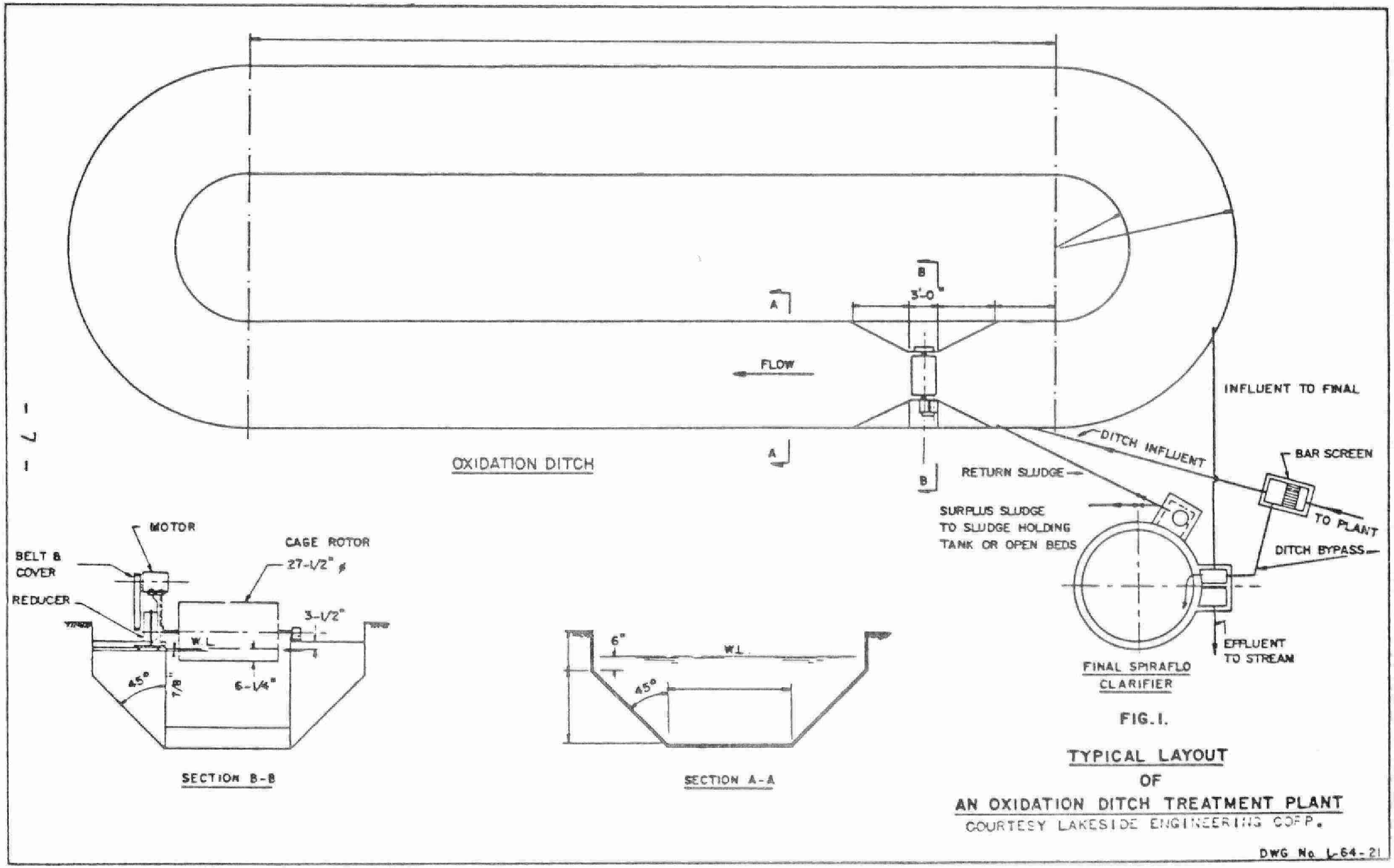

FIG. I.

TYPICAL LAYOUT OF AN OXIDATION DITCH TREATMENT PLANT

COURTESY LAKESIDE ENGINEERING CORP.

Fig. 2
CAGE ROTOR

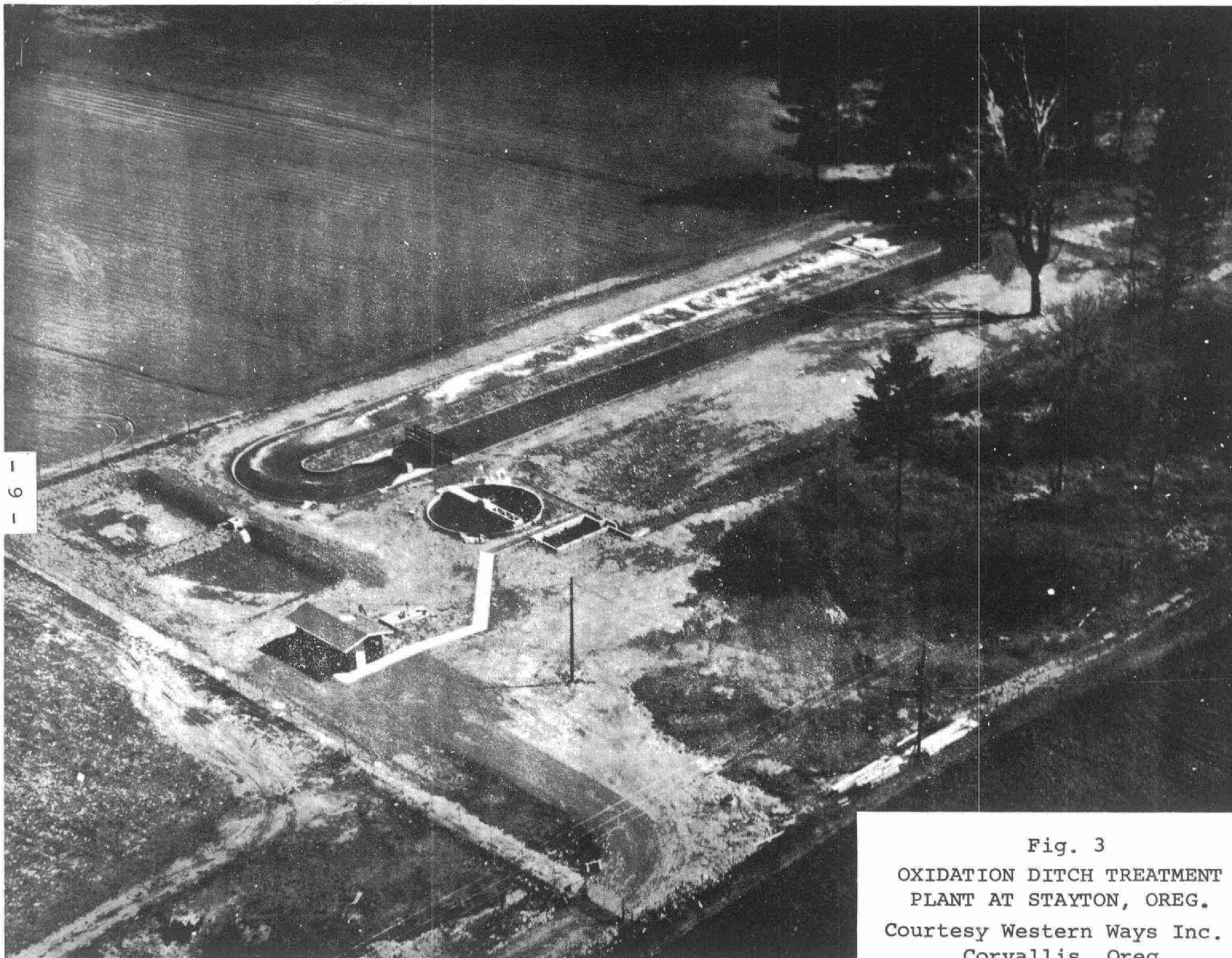

Fig. 3
OXIDATION DITCH TREATMENT
PLANT AT STAYTON, OREG.
Courtesy Western Ways Inc.
Corvallis, Oreg.

3. FIELD OBSERVATIONS

To obtain a more accurate picture of the actual operating conditions of a plant, it was decided to measure the dissolved oxygen content of the mixed liquor and the final effluent by means of a Pro-Tech Dissolved Oxygen Meter.

Determination of dissolved oxygen in the mixed liquor was limited by the 10-ft. long cord connecting the D.O. probe to the instrument itself. Where a walk bridge was laid across the ditch, measurements were always made at the centre of the ditch up to one foot below the liquid level. In the absence of a walk bridge, D.O. determinations were made from one of the sides. The rate of oxygen utilization of the mixed liquor was also measured, using the D.O. meter combined with a specially constructed cell. Samples for this determination were usually taken at a point up to 20 ft. downstream of the rotor, again depending on the physical layout of the ditch.

Assuming steady state conditions in the aeration system, the actual rate of aeration may be calculated from the rate of oxygen utilization, the dissolved oxygen and the temperature of the mixed liquor (4). Generally this rate of aeration will be less than the oxygenation capacity determined under similar conditions of rotor speed and immersion in tapwater at 0.0 mg/l D.O. and 20°C. It should be pointed out, that the steady state technique will at best give an approximation of the actual rate of aeration, mainly because operating conditions in an aeration system are not "steady state", but continuously varying throughout the day. Also the rate of oxygen utilization depends on the dissolved oxygen content of the mixed liquor, below a minimum value of 0.5 mg/l D.O. When the D.O. falls below 0.5 mg/l, the rate of oxygen utilization increases sometimes to double its value. This means that unreliable results may be obtained if the D.O. is less than 0.5 mg/l.

Other observations made during a plant visit consisted of the 30-minute settling test, measuring the speed of rotation and the immersion of the rotor, estimating the daily

plant flow, and a general visual inspection. Where available, plant operating records were analyzed.

In the following pages, the several installations are discussed in the order of the inspection tour. The discussion on each plant is subdivided into four sections:

1) description of plant layout and contributory population,
2) operation of the plant,
3) required maintenance and operator's time,
4) capital cost.

3.1 BEAVERTON, OREGON

Tektronix, Inc., treatment plant for the sanitary wastes of the company.

3.1.1 Description of Plant Layout and Contributory Population

The plant was installed for the treatment of the company's segregated sanitary wastes. In addition to the sanitary wastes from the industry, the oxidation ditch also receives waste water from an ice cream parlor, a shopping plaza, a bowling alley and a small trailer court. The trailer court provides a small but continuous load over the weekend, because the industry operates on a 5-day week basis.

Approximately 4,000 people are employed by Tektronix, most of whom work in the daytime only. Therefore the treatment process receives the major part of its organic loading between 8:00 a.m. and 5:00 p.m.

All flows arrive at the plant site by gravity, flowing into a pumping station. Originally the wastewater was pumped from here to the municipal treatment plant at Beaverton. Cost studies had shown that a separate oxidation ditch treatment plant would be more economical than pumping the sewage to Beaverton and paying the cost of treatment to the municipality.

The treatment plant was constructed on a tract of land adjacent to the pumping station and the Beaverton Creek. The plant consisted of an oxidation ditch, a secondary clarifier

and a chlorine contact basin. A flow meter was installed on a Parshall flume, through which the effluent from the clarifier flowed into the chlorine contact basin. The clarifier was built above grade. This made it necessary to pump the effluent from the ditch into the clarifier. The return sludge flowed by gravity into the ditch. The mixed liquor flowed by gravity from the ditch into the clarifier pump well. The gravity flows of both the mixed liquor and the return sludge could be controlled by means of adjustable telescopic valves. However the operation of the clarifier was in effect intermittent, because the feed pump was float-controlled. Due to the continuous withdrawal of return sludge the level in the clarifier dropped below the weir, whenever the feed pump was not operating.

Land area was available for conversion to sludge drying beds. When the plant was visited, there had been no need for regular sludge wasting during the first year of operation.

The raw wastes were pumped directly into the ditch without screening or comminution; the feed pipe terminated a few feet upstream of the east rotor in a tee.

The ditch was fully lined with a 2½ - 3½ inch thick layer of gunnite. Oxygen and circulation were provided by two cage rotors. They were mounted at opposite ends of the parallel channels of the ditch, and measured 10 ft. long and 27½ in. in diameter each. Each was driven at 75 rpm by a 7.5 HP motor.

Settled sludge from the clarifier returned to the ditch by gravity, entered also immediately upstream of the east rotor. The mixed liquor was discharged from the ditch through a 10-in. diameter gravity line to the clarifier pump well. The discharge line was connected to a shallow sump at the east end of the ditch.

The plant is equipped with a small but functional control laboratory, where routine control tests can be carried out, such as settleable solids, dissolved oxygen, BOD, pH, chlorine residual. The Company's chemical laboratory has facilities available for the determination of suspended solids and volatile solids, thus fairly complete plant operating records could be kept.

3.1.2 Operation of the Plant

The treatment process was put on stream in June 1963. Operating records were kept from July 1st on. As could be expected, an activated sludge developed in the aerated sewage. The mixed liquor suspended solids concentration was increasing gradually according to daily results of the 30-minute settling test. This was also indicated by suspended solids determinations. The MLSS increased from 2,000 mg/l in August to 2,500 in September to 4,000 in November and 4,900 mg/l in December. In February 1964, an MLSS content of 7,600 mg/l was observed. Some sludge was wasted in March, thereby reducing the MLSS to 6,700 mg/l. By May, the MLSS had increased again to 7,700 mg/l, one sample showing 8,100 mg/l.

The hydraulic load increased to a value of 130,000 gpd by the end of July 1963. A further increase was noted by October 1963, to a flow of approximately 170,000 gpd. The plant flow remained more or less constant at this level. The average daily flow was 170,000 gpd for the period of October to December.

During the plant inspection, conducted in the week of May 25, the daily flows varied as shown in Table I. Totalized flows were measured on a 24-hr. basis at approximately 1:30 p.m.

Table I

Daily Flow Through The Plant

Date		Volume (gallons)	Comments
Tuesday,	May 26	170,200	full load
Wednesday,	May 27	193,800	full load
Thursday,	May 28	130,400	skeleton staff only for inventory
Friday,	May 29	95,000	Memorial Day Holiday
Saturday,	May 30	83,400	

The full load flow rates compare well with the computed average daily flow of 182,000 gallons. The shutdown in plant operations on Thursday did not coincide with the 24 hour flow measuring period beginning at 1:30 p.m. the previous day and therefore the actual flow for this calendar day will be considerably lower than indicated. Because the flow meter is installed on the plant effluent, any volume discharged from the plant as waste sludge will not be recorded. Since sludge was wasted on Friday and Saturday, the corresponding flow data should be increased by about 12,000 gallons.

Due to the very high suspended solids content, the settleability of the mixed liquor had deteriorated considerably. When the plant was inspected, the mixed liquor solids occupied 98 per cent of the original volume after settling for 30 minutes. By means of intermittent sludge wasting over the period of May 29 to June 1, the MLSS were reduced to 5,900 mg/l. However, the settleability had not improved.

The BOD removal efficiency of the treatment plant may be estimated from the BOD results on samples of the raw sewage and the final effluent. All raw sewage samples are grab samples, whereas a continuous sampler was installed on the final effluent in October 1963, giving 24-hour composite samples. The BOD results on the raw sewage are summarized in Table II.

Table II

BOD of Raw Sewage Based on Grab Samples

Period	Number of Samples	BOD mg/l Low	BOD mg/l High	BOD mg/l Average
July - September 1963	27	84	347	210
October - December 1963	22	66	231	150
January - May 1964	39	44	228	137

The degree of treatment may be estimated from the final effluent BOD results on the 24-hour composite samples. In the period of October to December, 91 per cent of 22 samples showed BOD's less than 25 mg/l, and 68 per cent of the samples showed BOD's equal to or less than 15 mg/l.

From January to May a small improvement was observed in the BOD removal: 93 per cent of 41 samples showed BOD's equal to or less than 15 mg/l, and 71 per cent of the samples showed BOD's equal to or less than 10 mg/l. However in this same period, the average BOD of the raw sewage decreased somewhat as is shown in Table II.

In an attempt to determine the approximate detention time in the system based on the average daily flow, the volume of the oxidation ditch and of the clarifier were calculated. With a liquid level of 3 ft., the volume of the ditch is 182,200 gallons. When the level is raised one inch, the volume increases by 6,000 gallons. When the level is raised three inches to 39 inches, the volume in the ditch becomes 200,400 gallons. Under normal operating conditions, the liquid level may vary between 36 and 39 inches. It should be noted, that the maximum volume per foot of rotor, or $\frac{200,400}{20}$ = 10,020 gallons per foot is well below the limiting design value of 16,000 gallons per foot (5), required for lined ditches in order to maintain adequate circulation velocities and thus to prevent sedimentation of mixed liquor solids.

The approximate detention time, based on an average daily flow of 182,000 gallons and an average ditch volume of 188,000 gallons, may be calculated as $\frac{188,000}{182,000} \times 24 = 25$ hours.

The volume of the clarifier was calculated as 40,500 gallons. On the basis of the average daily flow, the detention time would be $\frac{40,500}{182,000} \times 24 = 5.4$ hours. However, for an accurate determination, the rate of sludge return should be estimated. The gravity return rate was controlled by a telescopic valve, which was not calibrated. Only when sludge was wasted on May 29th could the return rate be estimated by measuring the drop of the liquid level in the ditch. On this basis, the rate of sludge return may be assumed to be approximately equal to the daily flow through the plant. On several occasions the return rate appeared to be well in excess of the flow rate of final effluent from the clarifier. The actual detention time in the clarifier is reduced to 2.7 hours or less.

As an operating routine, the east rotor was run continuously and the west rotor was run from 10:00 a.m. to 4:00 p.m.

Initial dissolved oxygen measurements on Tuesday morning, May 26th, showed a dissolved oxygen content of only 0.2 mg/l upstream of the east rotor. The D.O. probe was held in the centre of the ditch from the walk bridge. Immediately downstream of the east rotor, a dissolved oxygen concentration of 2.2 mg/l was observed. At 10:00 a.m. the west rotor was turned on. About 10 minutes later, the D.O. upstream of the east rotor had increased to 0.5 mg/l, but by 11:00 a.m. it had fallen off to zero. At 11:10 a.m. the D.O. downstream of the east rotor was only 0.3 mg/l.

Because of the apparent oxygen deficiency of the system, it was recommended that the west rotor also be run continuously in an attempt to provide the maximum available quantity of oxygen to the mixed liquor.

However, a D.O. survey at 4:30 p.m. on May 26th failed to show a significant increase in dissolved oxygen throughout the ditch. A maximum D.O. concentration of 0.7 mg/l was observed. At 8:45 p.m. another D.O. survey was run. As before, measurements were made upstream and downstream of both rotors. Observed D.O.'s ranged from 2.9 to 3.3 mg/l. It was evident then that with a decrease in plant loading at the end of the working day, the system was able to recover completely from the lack of oxygen throughout the day.

Similar oxygen deficiencies were observed on May 27, June 4 and 5. On each of those days the treatment plant received the full daily flow. Dissolved oxygen concentrations varied between 0.0 and 0.5 mg/l. On June 4 a D.O. survey was conducted in the evening. The results showed again a significant recovery with D.O.'s ranging from 2.7 to 3.9 mg/l.

From these observations it may be deduced, that the operation of the treatment process may not be affected adversely by the daytime oxygen deficiency, because the system is able to maintain adequate concentrations of dissolved oxygen as soon as the organic load is reduced at the end of the working day (i.e. about 5:00 p.m.).

Dissolved oxygen surveys were also conducted on May 28, when the industry was virtually shut down except for a skeleton staff, and on May 29 when the plant was shut down for the Memorial Day holiday. The reduction in treatment plant loading

is only partly shown by the 24-hr. flow records as shown in Table I. Perhaps the sanitary wastes from the industry are low volume but high strength. It is interesting to note that on both days, a minimum D.O. of 4.0 mg/l was maintained with both rotors in operation. There was no appreciable decrease of dissolved oxygen throughout the day on May 28. In the morning of May 29, the D.O. in the ditch had increased to 5.8 mg/l. Since no increase in plant loading was anticipated, the west rotor was turned off until early Monday morning (June 1) when the full daily load could be expected again. About five hours later, the dissolved oxygen upstream of the east rotor had decreased to 2.2 mg/l. Downstream of the rotor, the D.O. was 3.3 mg/l. This indicated that a single rotor was able to maintain adequate dissolved oxygen levels under conditions of reduced plant load.

As part of several extended D.O. surveys dissolved oxygen concentrations were also determined in the return sludge, the clarified effluent and the effluent flowing through the Parshall flume. On May 27 the clarified effluent and the return sludge both showed 0.0 mg/l D.O., whereas the effluent in the Parshall flume contained 6.2 mg/l D.O. The observed increase in D.O. is the result of aeration occurring as the effluent drops through 9 ft. from the clarifier launder to the inlet channel of the Parshall flume. On June 4 the return sludge contained 0.0 mg/l D.O. and the clarified effluent 0.1 mg/l. These observations coincided with previously discussed oxygen deficiencies in the mixed liquor, and with the sludge blanket in the clarifier rising to within a few inches of the liquid surface.

During the period of reduced plant loading on May 28, the rate of sludge return was increased in an attempt to lower the sludge blanket in the clarifier. While this succeeded, there was no improvement in the dissolved oxygen content of the return sludge. The clarified effluent showed 0.3 mg/l D.O. On May 29 under continued reduced plant loading, the return sludge contained 1.4 mg/l and the clarified effluent 1.0 mg/l dissolved oxygen, compared with a mixed liquor D.O. of 5.4 mg/l (measured at the clarifier pump well). When the mixed liquor D.O. dropped to 2.0 mg/l five hours after the west rotor was turned off, the return sludge contained 0.3 mg/l and the clarified effluent 0.8 mg/l D.O.

The measured rates of oxygen utilization were low, varying between 6.9 and 23.4 mg/l/hr., in spite of the very high mixed liquor suspended solids concentration in excess of 7,000 mg/l. The observed utilization rates as well as the calculated rates of aeration at 0.0 mg/l D.O. are tabulated in Table III. Saturation values of D.O. in mixed liquor were assumed to be 90 per cent of tapwater saturation values due to the high MLSS.

Table III

Rates of Oxygen Utilization and Rates of Aeration

Date 1964		Test NO.	Temp. $^{\circ}C$	D.O. mg/l	Resp.Rate mg/l/hr.	C_i mg/l	C_i-C_l mg/l	K_La 1/hr.	Rate of Aeration mg/l/hr.
May 26,	2:50 pm	1	20.0	0.4	23.4	8.3	-	-	-
	9:10 pm	2	20.0	3.0	9.9	8.3	5.3	1.87	15.5
May 27,	10:10 am	3	17.5	2.7	6.9	8.7	6.0	1.15	10.0
	10:49 am	4	17.5	1.5	15.0	8.7	7.2	2.08	18.1
	11:28 am	5	18.1	1.75	9.6	8.6	6.85	1.4	12.0
May 28,	3:15 pm	6	18.1	4.3	8.1	8.6	4.3	1.88	16.2
	3:58 pm	7	18.1	-	7.2	8.6	-	-	-
May 29,	10:13 am	8	16.5	5.9	7.2	8.85	2.95	2.44	21.6
	1:54 pm	9	17.5	3.7	7.8	8.7	5.0	1.56	13.6*
June 4,	8:15 pm	10	19.5	3.7	10.8	8.35	4.65	2.32	19.4
June 5,	4:35 pm	11	19.0	0.3	16.7	8.4	-	-	-

* - West rotor was not operating.

Simultaneous measurement of the dissolved oxygen concentration showed that low D.O.'s, 0.3 mg/l, corresponded to relatively high utilization rates, 16.7 and 23.4 mg/l/hr. Conversely, high D.O. levels correctly indicated low utilization rates, 7.2 to 10.8 mg/l/hr., such as were observed during evening recoveries and the period of reduced plant loading.

The tabulated data in Table III demonstrate the absence of steady-state conditions in the aeration system. The average

calculated rate of aeration, excluding test No. 9, is approximately 16 mg/l/hr. at zero D.O. Rates of aeration were not calculated for tests No. 1 and 11, because the corresponding D.O. concentrations were less than 0.5 mg/l.

To compute the corresponding rate of aeration in tapwater, an average rotor immersion must be assumed, because liquid levels in the ditch varied throughout the day, and also because the liquid level was not measured for every determination of utilization rate. Based on actual level measurements, 37 inches appears to be a reasonable average, corresponding to a rotor immersion of 7 inches and a ditch volume of 188,000 gallons. For a rotor speed of 75 rpm the rate of aeration is 2.2 lb. O_2 per hour per foot of rotor, on the basis of aeration test data obtained at the Iowa State University (6). Since the ditch is equipped with 20 ft. of rotor, the theoretical rate of aeration would be $\frac{2.2 \times 20 \times 10^6}{188{,}000 \times 8.34} = 28$ mg/l/hr.

To compare the theoretical rate with the actual rate of aeration the effect of the mixed liquor composition on the oxygen transfer coefficient and on the oxygen saturation value should be recognized. Assuming a decrease of the transfer coefficient of 15 per cent as compared to tapwater, and a mixed liquor saturation value of 90 per cent of the saturation value in tapwater, the computed rate of aeration becomes $0.85 \times 0.90 \times 28 = 21$ mg/l/hr. It should be noted that the correction factors are assumed, and do not necessarily correspond to actual operating conditions. On the other hand, the average rate of aeration may be subject to correction due to the absence of steady state conditions.

Although the data on the composition of the raw sewage are provided only by grab samples rather than 24-hr. composite samples taken proportionally to the flow, it is felt that the daily organic loading can be approximated using the average of a large number of grab sample BOD's and the average daily flows corresponding to the days of sampling. On the basis of 39 grab samples over the period January to May 1964, the average raw sewage BOD in the daytime was 137 mg/l (from Table II). The average daily flow in the same period was 182,000 gpd. The average organic loading would be $\frac{137}{10^6} \times 182{,}000 \times 8.34 = 208$ lb. BOD/day.

Design practice in the Netherlands (2) has been to provide 2 lb. O_2/lb. BOD applied, based on tapwater aeration test data. The oxygenation capacity (OC) of the aeration system will be 2.2 x 20 x 24 = 1,056 lb. O_2/day. The ratio OC/load becomes $\frac{1056}{208}$ = 5 lb. O_2/lb. BOD applied. In spite of the apparently adequate OC of the system, a definite oxygen deficiency was observed during the daytime periods of the week days. It is possible that grab samples do not provide an accurate picture, although sampling was always carried out in the daytime. Due to the uneven distribution of plant loading over a 24-hr. period, the actual daytime loading could be considerably higher, thereby decreasing the daytime OC/load ratio.

Adopting an OC/load ratio = 2 as design criterion allows for a 25 per cent reduction in oxygen transfer when aeration of mixed liquor is compared with tapwater aeration tests. Specific plant operating conditions may affect this reduction. It is possible that the actual reduction in oxygen transfer was greater than the allowed 25 per cent due to the very high solids content of the mixed liquor.

Assuming that the high MLSS concentration acts as a physical hindrance to oxygen transfer, it may be supposed that a satisfactory dissolved oxygen concentration can be maintained when the MLSS are reduced to a more normal range of 3,500 to 4,000 mg/l by controlled sludge wasting.

3.1.3 Maintenance

During the past year much difficulty has been experienced with mechanical maintenance of the aeration equipment. Weak points were the speed reducers, rotor bearings and rotor shaft. Size 1 speed reducers had been installed, but these are suitable for a uniform load only, whereas the actual load may be considered as an intermittent shock load resulting from the action of the rotor blades on the liquid. Size 2 speed reducers are specially designed for intermittent shock-loads and will be installed, replacing the present size 1 speed reducers.

The rotors were equipped with non-slip gear-type belt

drives. These belts were subject to repeated breakage. It was decided to replace these with regular V-belts to provide the required amount of slippage.

The original design of the rotor shafts was also found to be inadequate, resulting in shearing of the shaft at the rotor. By a trial and error procedure it was learned that the solid stub shafts on each end of a rotor should enter the hollow rotor shaft well beyond each end plate of the cage.

On the basis of these maintenance problems, the equipment manufacturer has revised the design of the rotors and has agreed to replace the equipment.

The operating time required is about 12 man-hours per week. The operator determines the BOD content of the raw sewage and the final effluent twice weekly besides the routine plant maintenance duties. In addition to the above operating procedure, the telescopic overflow valves of the ditch effluent and the return sludge are checked once every shift by a member of the industry's maintenance crew. There is a tendency for rags to collect at the top of the valves and to interfere with the flow of liquid.

3.1.4 Capital Cost

A figure of $66,000 was given as the capital cost of the installation. Included in this amount are the ditch, rotors, clarifier, chlorinator and contact basin, control room and the fence. The plant was constructed on a four-acre tract of land which was the property of the industry. Therefore, the cost of land was not included in the capital cost. Neither were the collection system and pumping station included.

3.2 CITY OF STAYTON, OREGON

Municipal waste treatment plant.

3.2.1 Description of Plant Layout and Contributory Population

The oxidation ditch treatment plant was installed to treat the municipal waste water of the City of Stayton. The estimated present population is 2,200, the plant having been designed for an ultimate population of 3,500. The design average daily flow was estimated at 430,000 gpd. The plant loading in terms of 5-day BOD was estimated at 630 lb. per day.

The treatment plant consists of a comminutor, an oxidation ditch, a clarifier and a chlorine contact chamber equipped with a 90° V-notch weir flow meter. The chlorinated effluent flows through an outfall ditch into the North Santiam River.

Raw sewage flows into the plant by gravity continuously. It passes through the comminutor and flows into the oxidation ditch just upstream of the north rotor. Adjacent to the raw sewage inlet is the return sludge inlet. Mixed liquor flows by gravity from the north end of the ditch into the clarifier. The liquid level in the ditch is fixed by the position of the peripheral weir of the clarifier. A skimmer continuously removes scum from the clarifier surface. The collected scum is returned to the ditch. Clarified effluent flows by gravity into the chlorine contact chamber, and passing over the V-notch weir into the outfall ditch. Settled sludge is returned to the ditch by means of return sludge pumps. To waste sludge the same pumps can pump settled sludge from the clarifier to sludge drying beds also located on the plant property.

The oxidation ditch is lined with a three-inch thick layer of shotcrete reinforced with wire mesh. Two cage rotors are installed at opposite ends of the parallel channels of the ditch. Each rotor measures ten feet long and 27½ inches in diameter, and is driven by a 7½ HP motor at 85 rpm and 7 inches immersion.

A control building contains the necessary electrical controls and chlorinating equipment, as well as a small laboratory.

Routine control tests such as settleable solids, residual chlorine, etc., can be conducted. At the time of the inspection, June 1-3, 1964, the laboratory had not been set up as yet. The city engineer will be assisted by the State Sanitary Authority in the selection and performance of desirable control tests.

3.2.2 Operation of the Plant

The treatment plant was put into operation in the fall of 1963. As more homes in the municipality were connected to the sewer system, the plant loading increased. By the beginning of June, 1964, approximately one half of the homes were connected. Operating records showed an average daily flow of 147,000 gallons. There were no data available on BOD, MLSS, percent settleable solids and dissolved oxygen. Presumably as a result of frothing during the early stages of operation, both rotors were operated intermittently on an hourly cycle, e.g. 30 minutes on and 30 minutes off.

A preliminary settling test, June 1st, a.m., showed 9 per cent settleable solids after 30 minutes. The supernatant liquid was cloudy. Overall appearance was poor quality, although the sludge was light brown. At the time of sampling the north rotor was in operation, the sample being taken from the centre of the ditch from the walk bridge downstream of the rotor. The very small volume of settleable solids was remarkable considering the length of time the plant had been in operation. In the afternoon of the same day another settling test showed 18 per cent settleable solids, this time both rotors were operating. Subsequent tests confirmed that both rotors were required to maintain the mixed liquor solids in suspension. When one rotor was turned off, the settling test showed approximately one half the settleable solids.

The volume of the ditch amounted to 352,000 gallons. The total rotor length was 20 ft., or 17,600 gallons per foot. The recommended maximum for lined ditches is 16,000 gallons per foot of rotor. It was also observed that the rotors were striking the liquid with the wrong side of the blades, i.e., the first part to come in contact with the liquid was the web of the T-shaped rotor arms, rather than the flat blades. This condition may reduce the propelling force of the rotors.

When both rotors were operating on the one-hourly cycles, a dissolved oxygen check was made at a point downstream of the north rotor from the walk bridge in the centre of the ditch to demonstrate the variation of dissolved oxygen with rotor operation. The results were:

the D.O. was 1.5 mg/l with both rotors on,
the D.O. was 1.0 mg/l with north rotor on and south rotor off,
the D.O. was 0.5 mg/l with south rotor on and north rotor off,
the D.O. was 0.0 mg/l with both rotors off.

Also on the basis of these observations, it seemed necessary to operate both rotors continuously.

On June 2nd and 3rd, several D.O. surveys were conducted on the aeration system. The rotors were running continuously since 4:00 p.m. June 1st. The observed variation in dissolved oxygen throughout the ditch was fairly consistent. Generally a D.O. between 1.0 and 1.2 mg/l was found a short distance downstream of the north rotor, and downstream of the south rotor the D.O. ranged between 0.6 and 1.0 mg/l. Most D.O. measurements at the upstream ends of the ditch ranged from 0.0 to 0.2 mg/l. One of the surveys was run in the evening starting at 7:50 p.m. There was no significant change in the distribution of dissolved oxygen in the system.

In the afternoon of June 2nd, a D.O. survey was made about one hour after the south rotor had been turned off. A decided improvement was observed. Downstream of the north rotor, the D.O. was 1.5 mg/l, downstream of the south rotor 0.8 mg/l. At the north and south upstream ends D.O.'s of 1.0 and 0.7 mg/l were observed. Since part of the mixed liquor solids was not kept in suspension, the oxygen demand of the mixed liquor was reduced, and therefore the D.O.'s increased even though the supply of oxygen was reduced by 50 per cent.

The D.O. of the clarified effluent was measured twice. On June 1st under conditions of intermittent operation of the rotors, a D.O. of 1.0 mg/l was observed. In the late afternoon of June 3rd, the clarified effluent contained 0.0 mg/l D.O. The observed change in dissolved oxygen may have resulted from the changeover to continuous operation of the rotors.

Also in the late afternoon of June 3rd, a D.O. survey was made across the west channel of the oxidation ditch downstream of the north rotor from the walk bridge. Results are given in Table IV.

Table IV

Distribution of Dissolved Oxygen Across the West Channel

Distance from west edge, feet	3'	6'	9'	12'	15'	18'
D.O. mg/l	0.0	0.3	0.6	0.3	0.0	0.0

A rather marked variation of dissolved oxygen was observed, increasing from 0.0 mg/l near either side of the channel to a maximum concentration in the centre. It was not possible to determine the vertical distribution of D.O. Perhaps the observed variation was again the result of the incorrect mounting of the rotors.

Several tests were carried out to determine the rate of oxygen utilization of the mixed liquor. The results are summarized in Table V.

Table V

Rates of Oxygen Utilization and Rates of Aeration

Date 1964	Test No.	Temp. °C	D.O. mg/l	Resp.Rate mg/l/hr.	C_i mg/l	C_i-C_l mg/l	K_{La} 1/hr.	Rate of Aeration mg/l/hr.
June 1								
3:00 pm	1	18.0	1.8	10.8	9.05	7.25	1.49	13.5
3:50 pm	2	18.0	2.2	7.8	9.05	6.85	1.14	10.3
June 2								
10:15 am	3	15.0	1.0	7.5	9.65	8.65	0.87	8.4
2:15 pm	4	16.5	1.0	9.0	9.35	8.35	1.08	10.1

For these tests the dissolved oxygen saturation value was assumed 95 per cent of the tapwater saturation value. The observed

utilization rates were again low, indicating a low organic loading. Tests No. 1 and 2 were run during the intermittent operation of the rotors: for test No. 1 both rotors were running, for test No. 2 the south rotor had been turned off for 50 minutes. Previous observations on settleable solids and dissolved oxygen were confirmed by the rates of oxygen utilization: higher D.O. and less settleable solids corresponded to a lower utilization rate.

Only the results of tests No. 3 and 4 can be used for estimating the actual rate of aeration, because the aeration system was assumed to be stabilized under continuous operation of the rotors for at least 18 hours. The average estimated rate of aeration is 9.2 mg/l/hr. in the mixed liquor at 0.0 mg/l D.O.

On the basis of tapwater aeration test data, the oxygenation capacity of the system can be calculated. For an immersion of 6½ in. and a speed of rotation of 85 rpm, a 27½-in. diameter cage rotor will dissolve 2.6 lb. O_2 per hour per foot of rotor (6). However, since the rotors were mounted incorrectly, the OC will be reduced to 82 per cent (7), i.e. 0.82 x 2.6 = 2.13 lb. O_2/hr./ft. of rotor. This value of OC is equivalent to $\frac{2.13 \times 20 \times 10^6}{352,000 \times 8.34} = 14.5$ mg/l/hr.
Assuming that the oxygen transfer coefficient is reduced to 85 per cent, when aeration occurs in mixed liquor, and that the oxygen saturation value in mixed liquor is 95 per cent of the tapwater value, the available rate of aeration can be estimated at 0.85 x 0.95 x 14.5 = 11.7 mg/l/hr. This figure compares reasonably with the rate of aeration estimated from the rates of oxygen utilization.

In the absence of BOD results for the raw sewage as well as the clarified effluent, the design criteria must be used to arrive at approximate values for the OC/load ratio. At the time of the plant inspection the average flow was 147,000 gpd, corresponding to a contributory population of $\frac{147,000}{123}$ = 1,200, and to an average BOD loading of 1,200 x 0.18 = 216 lb. BOD per day. The tapwater OC is 2.13 x 20 x 24 = 1,022 lb. O_2 per day. The OC/load = $\frac{1022}{216}$ = 4.7 lb. O_2/lb. BOD applied.

It is possible, that the actual BOD load was greater than 216 lb. per day, since the per capita flow rate may have been less than 123 gpd. This supposition can only be confirmed by BOD analyses on 24-hr. composite samples of the raw sewage.

The OC of the aeration system will be 2.75 lb. O_2/hr./ft. of rotor when the rotors are correctly installed, operating at an immersion of 7 inches. The design BOD loading is estimated at 630 lb. per day. The ultimate OC/load ratio will be $\frac{2.75 \times 20 \times 24}{630} = 2.1$ lb. O_2/lb. BOD applied. Thus the aeration system appears adequate for the design load.

3.2.3 Maintenance

No mechanical breakdowns have been experienced with the equipment so far. It should be noted, that size 2 speed reducers were used for the aeration rotors. Since both rotors were mounted incorrectly when the plant was constructed, the equipment supplier has agreed to replace the rotors in the fall of this year.

The time required to operate the plant will be approximately one man-hour daily, including routine tests in the control laboratory.

It would appear to be a drawback, that the immersion of the rotors cannot be adjusted, i.e., that the oxygenation capacity of the system is a fixed quantity. Another drawback of the plant design is the inaccessibility of the return sludge, as it is pumped from the clarifier to the ditch. The only method of checking the quality of the return sludge is by wasting some to the drying beds.

3.2.4 Capital Cost

The construction of the plant cost $94,000.00. This amount does not include the collection system and a manhole on the plant property. An additional amount of $10,000.00 was required for ten acres of land, of which nearly two acres were needed for a 2,800 ft. long access road. The remaining eight acres were used as plant site, although the fenced-in part of the site, on which the plant was built, occupied only three acres.

3.3 VILLAGE OF WILLIAMS LAKE, B.C.

Municipal Waste Treatment Plant.

3.3.1 Description of Plant Layout and Contributory Population

The treatment plant was built to treat the domestic wastewater from the south half of the town, replacing a municipal septic tank. The plant was located adjacent to the site of the septic tank.

The treatment plant was designed for an estimated population of 2,000 with a per capita flow rate of 90 gpd (75 Imp. gpd).

It was interesting to note that the oxidation ditch also served as a clarifier, due to intermittent operation of the rotor. This was the only plant of the intermittent type visited during the Inspection Tour.

In an intermittent system wastewater cannot flow through the plant continuously. The original design, developed in the Netherlands (1), included an aeration period of 4½ hr. followed by a settling period of 1½ hr. Only during the last part of the settling period was sewage admitted to the ditch, displacing the clarified treated effluent over the discharge weir. With such a schedule of operation, sewage was stored in the wet well and the sewer system for a maximum of 5½ hr.

At Williams Lake the design was considerably different, because the overflow device discharged only the upper 4 inches of liquid from the ditch after each settling period. The volume of the intermittent discharge was approximately equal to 9 per cent of the volume of the ditch. Assuming that the daily flow is equal to the ditch volume, the operating cycle of the plant must be repeated eleven times per 24 hr.

Because of the intermittent operation of the rotor, the abandoned septic tank was used as a holding basin for storage of the raw sewage flow. A timer-operated control valve allowed the required volume of sewage to flow into the ditch after the clarified liquid had been discharged.
Aeration and circulation were provided by a 10-ft. long, 27½-in. diameter cage rotor driven by a 10 HP motor.

The clarified effluent was discharged into a nearby watercourse. A small building housed the motor drive and speed reducer of the rotor, as well as the necessary electrical controls.

Only the sides of the ditch in the bends were concrete-lined. Prior to and during construction of the plant several modifications had been made, so that the present ditch did not correspond exactly to available drawings. The volume of the ditch was estimated at approximately 192,000 gallons (160,000 Imp. gallons). The plant was not equipped with a course screen or rack, a flowmeter and a chlorinator. A barminutor was to be installed later this year, in an attempt to reduce operating problems caused by rags.

3.3.2 Operation of the Plant

The treatment plant was put into operation on November 7, 1963, by filling it with the contents of the town septic tank. The extreme quality of that liquor was clearly indicated by a grab sample of the plant effluent, taken at 10 a.m. the same day, showing a BOD of 1325 mg/l. An effluent grab sample on November 27, 1963 contained a BOD of 85 mg/l, showing a significant improvement in the operation of the treatment process. A third effluent sample was taken on March 18, 1964; this contained 65 mg/l BOD. These data give only an indication of the extent of treatment, because influent BOD loadings are not available.

The plant was inspected on June 9, 1964. The overall impression was that the process was of a very poor quality. A characteristic odour of oxygen deficiency was noticed. Gas bubbles were rising to the surface in certain sections of the ditch, indicating the presence of septic sludge deposits. The mixed liquor solids appeared grey. A 30-minute settling test showed only 7 per cent settled solids by volume, the supernatant liquid contained significant quantities of floating and suspended solids as well as turbidity.

A dissolved oxygen survey, conducted from 10 a.m. on, showed 1.0 mg/l D.O. a short distance downstream of the rotor and 0.0 to 0.2 mg/l upstream of the rotor. At the start of the aeration period at 9:50 a.m. the operating schedule was

changed so as to increase the aeration period to two hours and to reduce the settling period from 50 to 30 minutes. The dissolved oxygen concentrations did not change in the course of the aeration period. A D.O. survey conducted in the evening about 7:45 p.m. showed a similar D.O. pattern as observed earlier.

It should be noted that operating experience in the Netherlands (1) with an intermittent oxidation ditch showed a gradually increasing D.O. content of the mixed liquor during a 4½ hr. aeration period. Maximum values of 3 mg/l were observed. The lack of increase of the dissolved oxygen concentration over a 2-hr. aeration period would indicate that the oxygenation capacity of the system is inadequate for the present loading. The OC might be increased by lengthening the aeration period above 2 hr. However, this would necessitate a complete modification of the effluent overflow device to permit a greater volume to be discharged after each settling period. Ideally the overflow device should discharge 1/4 of the average dry weather flow, resulting in only four operating cycles per day and therefore a maximum OC of the system.

Two tests were run to determine the rate of oxygen utilization of the mixed liquor. On both occasions samples were taken after identical fractions of the 2-hr. operation period. Values of 14.7 and 19.5 mg/l/hr. were observed, the higher value for a sample taken at 8:10 p.m. The results suggested a higher oxygen requirement than could be expected from the observed quantity of settleable solids in the mixed liquor, and also higher than utilization rates observed at other oxidation ditch installations.

The observed operating conditions may have been the result of the particular start-up procedure followed by inadequate aeration until the date of the plant inspection. It is likely that the deposits of septic sludge were partly the result of the start-up procedure. However, the available rotor length was inadequate for the ditch volume to provide the required circulation. The recommended ditch volume per unit rotor length in an unlined ditch is 13,000 gallons per foot of rotor (10,800 Imp. gallons) (5). The actual ratio was calculated as 19,200 gallons per foot of rotor.

From the measured oxygen utilization rates the oxygenation capacity of the rotor may be estimated at zero mg/l D.O. in the mixed liquor. An average value of 19.5 mg/l/hr. was calculated, corresponding to 31 lb. O_2/hr. Although this figure might be subject to some error, as a result of the low average dissolved oxygen concentration of the mixed liquor and of the lack of increase of D.O. over the aeration period, it might be considered an approximate value and can therefore be used in the following discussion. The daily BOD loading may be estimated at 2,000 x 0.17 = 340 lb. To satisfy this demand the rotor should supply the required amount of oxygen during the total aeration periods rather than a 24-hr. day. For the extended aeration period of 2 hr. the actual OC will be $\frac{2.5}{3} \times 31 \times 24 = 620$ lb. O_2 per day. The OC/load ratio becomes $\frac{620}{340} = 1.8$ lb. O_2 per lb. BOD applied. Thus it appears that a 2-hr. aeration period may give satisfactory operation after steady state operating conditions have been achieved. However, in view of the above reported D.O. surveys, and because of the various assumptions for the calculated OC, it seems necessary to increase the aeration period in an attempt to satisfy the present oxygen requirement. An increased OC would facilitate a greater BOD removal, resulting in a build-up of the mixed liquor suspended solids.

3.3.3 Maintenance

No mechanical breakdown had been experienced during the first seven months of operation. About one man-hour daily was required for routine operating procedure and maintenance.

It was interesting to note, that during last winter's cold spell no special precautionary measures were needed and that no operating problems were encountered. There was no ice build-up on the rotor, even with intermittent operation. For a week low temperatures of 30 below zero ^{o}F were recorded with corresponding daytime temperatures of zero ^{o}F. For a period of a month low temperatures of about zero ^{o}F were observed. Weather of this severity may occur in many areas of Ontario.

3.3.4 Capital Cost

The cost of installation of the treatment plant amounted to $30,000. The land was not included, because it is leased. The plant is financed through a CMHC village by-law over a period of 20 years.

3.4 VILLAGE OF MONTROSE, B.C.

3.4.1 Description of Plant Layout and Contributory Population

The treatment plant was built to treat the domestic wastewater from the village, replacing the individual home septic tanks. The plant was constructed on a narrow ledge of the hillside at an elevation of approximately 100 feet less than the average elevation of the village. An oxidation ditch was preferred over a lagoon, because available land area was the controlling factor.

The design population was estimated at 1,500 with a per capita flow rate of 96 gpd (80 Imp. gpd). The present population was estimated at 950. It was interesting to note, that the ditch was designed and constructed for the design population of 1,500, but that the layout was modified to be adequate for the present loading conditions by shortening the length of the ditch. Thus the capacity of the oxidation ditch was reduced from 142,000 gallons to 94,000 gallons (78,400 Imp. gallons). The design loading in terms of 5-day BOD may be estimated at 1,500 x 0.17 = 255 lb. BOD per day.

The treatment plant consists of a coarse screen, an oxidation ditch, a hopper-bottom Spiraflo clarifier, and a chlorine contact chamber.

Raw sewage flows into the plant by gravity, passing through a coarse screen and through an 8-in. diameter pipe into the ditch about ten feet upstream of the rotor. Aeration and circulation of the mixed liquor are provided by a 6-ft. long 27½-in. diameter cage rotor, rotating at 75 rpm driven by a 5-HP motor. The immersion of the rotor can be varied by adjustment of the effluent weir trough, which is suspended from a bridge across the clarifier by adjustable bolts. The mixed liquor flows from the ditch by gravity through a submerged 8-in. diameter pipe into the clarifier. Settled sludge is pumped to the intake works, where it is mixed with the incoming raw sewage ahead of the coarse screen. Clarified effluent passes through a fully covered chlorine contact tank, and is discharged into the Beaver Creek. Sludge drying beds had not been constructed, although there were some vacant areas inside the fenced-in plant property, which could perhaps be turned into drying beds when necessary.

A small building was constructed beside the rotor on the outside of the ditch. It contained the various electrical controls, the chlorinator and also the motor, drive and speed reducer of the cage rotor. Minimal facilities were provided for routine control tests such as relative stability and residual chlorine.

3.4.2 Operation of the Plant

The treatment system was put into operation in September 1963. Gradually the plant loading increased as the houses in the village were connected to the collection system, and the domestic septic tanks were emptied into the sewer. In the early stages of operation serious trouble was experienced with frothing due to the initially low organic load. Perhaps as a result of this the rotor was operated intermittently.

The plant was inspected on June 11 and 12, 1964. At that time the schedules of operation of the aeration rotor and the return sludge pump were intermittent as follows: the rotor operated for 15 minutes, during which time the return sludge pump was at rest; when the rotor stopped, the pump started up and ran for 15 minutes, until the rotor started operating again and the cycle was repeated. The results of this procedure were an oxygen deficient, poor quality mixed liquor, deposits of septic sludge on the bottom of the ditch and in the clarifier, both indicated by gas formation and a poor quality, highly turbid effluent. To indicate qualitatively the turbidity of the effluent, a reflecting surface disappeared from sight only 5 inches below the liquid surface in the clarifier. The deposits of septic sludge may have originated from the draining of the individual septic tanks, when the homes were connected to the sewer system. The suspected oxygen deficiency was confirmed by a measured dissolved oxygen concentration of 0.2 mg/l downstream of the rotor from the walk bridge during the 15 minute aeration period.

Continuous operation of both the rotor and the return sludge pump was recommended as essential for satisfactory operation of the treatment system. The intermittent schedule of operation was discontinued on June 11 at 2:30 p.m. By 5:15 p.m. a small improvement was observed in the D.O. content of the mixed liquor to 0.5 mg/l.

There were no operating records of the plant. Since the plant was not equipped with a flowmeter, the actual flow through the plant could only be estimated from water consumption records accumulated during the winter months. The average daily water consumption over a period from December 1st, 1963 to March 31st, 1964 was 49,600 gallons (41,400 Imp. gallons). For a population of 950 this amounted to a per capita flow rate of 52.2 gpd (43.5 Imp. gpd) as compared with a design rate of 96 gpd. Therefore the retention time of the ditch was $\frac{94,000}{49,600} \times 24 = 45.5$ hr. rather than a design retention time of $\frac{94,000}{950 \times 96} \times 24 = 24.7$ hr.

From several settling tests it could be concluded that the rotor was able to keep the settleable solids in suspension during intermittent operation. A settling test taken during a 15 minute aeration period showed 22 per cent settleable solids, compared with an average 19 per cent from three tests taken the following day after a minimum 19 hours of continuous operation of the rotor. The observed quantity of settleable solids indicated a fair build-up of mixed liquor solids. However, if the treatment process had effected a greater removal of organic material, the amount of settleable solids would have been greater due to cell synthesis. Although no BOD analyses were available, a qualitative inspection of the clarified effluent suggested that BOD removal was far from complete, thus keeping cell growth to a minimum.

Two D.O. surveys were carried out on June 12th. Downstream of the rotor at the walk bridge dissolved oxygen concentrations of 1.4 and 1.2 mg/l were measured. At the upstream end the corresponding D.O.'s were 0.0 and 0.5 mg/l. Continuous operation of the rotor resulted in a significant improvement of the mixed liquor dissolved oxygen content.

The rate of oxygen utilization of the mixed liquor was also measured on two samples taken in the afternoon of June 12th. Values of 11.7 and 17.5 mg/l/hr. were observed. The wide difference between the two results, combined with the change from intermittent to continuous operation on the previous day, did not warrant the calculation of rates of aeration at zero mg/l D.O. in the mixed liquor.

Measurements showed that the rotor operated at an immersion of 10 inches. The oxygenation capacity at this immersion is 3.0 lb. per ft. of rotor per hr. This is equivalent to 3.0 x 6 = 18 lb. O_2/hr. or $\frac{18 \times 10^6}{94,000 \times 8.34} = 23$ mg/l/hr. When the effects of mixed liquor on aeration are taken into account, the rate of aeration at 0.0 mg/l D.O. becomes 0.85 x 0.95 x 23 = 18.5 mg/l/hr. It is significant that this value exceeds the measured rates of oxygen utilization.

Since the net power required to operate the rotor at 75 rpm and an immersion of 10 inches is 4.9 HP (6), the liquid level in the ditch should be lowered to prevent overloading of the drive motor. If the immersion is decreased by 2 inches, the oxygenation capacity will be reduced to 2.5 lb. per ft. of rotor per hr. The available rate of aeration will then be $\frac{2.5}{3.0} \times 18.5 = 15.4$ mg/l/hr., which may be expected to be adequate for the oxygen requirements of the process, once the ill effects of the previous intermittent operation have been overcome. Under continuous aeration the rate of oxygen utilization will decrease to a value less than 15 mg/l/hr., also as a result of removing the deposits of septic sludge from the bottom of the ditch.

The OC/load ratio can be calculated from the BOD design criteria. The present population of 950 is assumed to contribute a load of 950 x 0.17 = 161.5 lb. BOD per day. At an immersion of 8 inches the OC = 2.5 x 6 x 24 = 360 lb. O_2 per lb. BOD applied. When the BOD load increases to the design load, the OC/load ratio becomes $\frac{360}{1500 \times 0.17} = 1.41$ lb. O_2 per lb. BOD applied. Therefore rotor length must be increased to meet the increased oxygen requirements.

A 6-ft. rotor will provide adequate circulation in the shortened ditch, as shown by the ditch volume per foot of rotor: $\frac{94,000}{6} = 15,600$ gallons per foot of rotor. The suggested maximum is 16,000 gallons (or 13,300 Imp. gallons) per foot of rotor. However, when the ditch is extended to its full use, additional rotor length must be installed to keep the ditch volume per foot of rotor below 16,000 gallons. The volume of the ditch is increased by 48,000 gallons. The extra

rotor length will be $\frac{48,000}{16,000}$ = 3 feet. Therefore the oxygenation capacity will be increased by 3 x 2.5 x 24 = 180 lb. O_2 per day, giving an ultimate OC/load ratio of $\frac{360 + 180}{1500 \times 0.17}$ = 2.12 lb. O_2 per lb. BOD applied. This should be adequate for efficient operation of the treatment process.

3.4.3 Maintenance

Some difficulties were experienced with the rotor drive mechanism. A so-called gear belt drive was used, but the belt broke on several occasions. It was felt that the positive drive was unsatisfactory for this particular application, and that it should be replaced with a regular V-belt, thus allowing some slip between the motor and rotor shafts.

The time needed for routine operation of the plant was approximately one man-hour daily. Routine duties included manual cleaning of the coarse screen, and scraping down of the hopper bottom of the clarifier to prevent the accumulation of sludge. The gravity line, through which the raw sewage and return sludge flow from the intake works to the ditch, clogs occasionally, presumably as a result of solids settling out in the line during periods of low flow. This may be considered a drawback of the design of the plant.

3.4.4 Capital Cost

The plant was constructed at a total cost of $31,000 being financed by a bond issue over a period of twenty years. The land area and the collection system were not included.

3.5 CITY OF REGINA, SASKATCHEWAN

Pilot plant for the treatment of municipal wastewater.

3.5.1 Description of the Plant Layout

The pilot scale oxidation ditch formed part of a long term research program, undertaken by the City of Regina to determine a suitable means of treatment to replace and extend the existing overloaded municipal treatment plant.

Two unused aeration tanks, each measuring 82 ft. long by 9½ ft. wide by 11 ft. deep, were converted to an oxidation ditch-type circuit (8,9). Two sections of the common wall were removed one at each end of the tank. Both ends of the combined tanks were built in with concrete at a 10-ft. radius to prevent sludge deposition in the corners. An average liquid level of 4 feet was maintained in the ditch. A 3-ft. long 27½ in. diameter cage rotor was mounted in one tank about 6 feet below the top of the tank wall. The volume of the ditch was 48,000 gallons (40,000 Imp. gallons) for a 4-ft. liquid level.

A hopper-bottomed settling tank was built in a third aeration tank. The settling tank measured 9 ft. 4 in. long by 9 ft. 2 in. wide and 9 ft. 3 in. deep. The walls of the hopper section were sloped at a 60° angle. An overflow weir was installed along one side only.

The pilot plant feed stream was taken from the main plant flow after comminution and grit removal. It entered the oxidation ditch by gravity from a weir box. In this way the flow rate could be set and kept constant at certain selected levels. The mixed liquor was pumped from the ditch into the settling tank. The settled sludge returned to the ditch by gravity. The clarified effluent discharged over the weir through a channel into a barrel, from where it was pumped through a water meter for flow measurement.

3.5.2 Operation of the Plant

The pilot plant was put into operation in March 1963, and was in continuous operation till July 1964. Due to the

nature of the project detailed operating records were kept. Attention was given not only to BOD removal and suspended solids reduction, but also to the concentration in the effluent of free ammonia and nitrates.

The experimental work was carried out in three main stages, the retention time in the ditch being the major variable. Retention times of 2.7, 1.5 and 1.0 days respectively were selected. Experimental conditions were held essentially constant over a period of three to four months, so as to achieve steady operating conditions. Ambient conditions could not be controlled.

Operating results showed BOD removal efficiencies ranging from 87 to 96 per cent producing effluents with BOD concentrations from 15 to 30 mg/l. These data were obtained in the period of July to October, 1963, when the retention time in the ditch was kept constant at approximately 2.7 days. During the winter period from November 1963 to February 1964, the daily flow was increased, reducing the retention time to 1.5 days. The BOD removal efficiency varied between 80 and 88 per cent; most of the effluent BOD results were in the range of 30 to 40 mg/l. With a further decrease of retention time to 1.0 day at the end of February 1964, BOD removal efficiencies remained of the same order of magnitude as observed during the winter period.

However, BOD removal improved in the first half of May, resulting in effluent BOD's of 10 to 15 mg/l which corresponded to removal efficiencies of the order of 95 per cent. It should be noted that through the second half of May and the month of June the effluent quality deteriorated somewhat showing a small increase in BOD to 25 and 35 mg/l.

Until early December 1963 significant concentrations of nitrites were found in samples of the clarified effluent, ranging from 2 to 25 mg/l. In the same period the corresponding free ammonia concentrations of the effluent reached a maximum of 14 mg/l. From then on, a marked reduction of nitrites occurred to 0 and 3 mg/l. The free ammonia content increased to 25 mg/l. It is possible that the higher NH_3 concentrations caused the BOD of the effluent to increase, provided that nitrifying organisms were still present.

In this connection it is of interest to note how Pasveer (1) determined an overall BOD on effluent samples, and by pasteurization and seeding determined the non-ammoniacal BOD. In most samples with an appreciable NH_3 content the pasteurized fraction showed a significantly lower BOD.

Mixed liquor suspended solids were generally kept at approximately 6,000 mg/l. To achieve it, regular wasting of excess sludge was necessary. In spite of the relatively high MLSS, there was no indication of sludge deposits in the ditch. The rotor provided adequate circulation to maintain the solids in suspension. When the ditch was operated at the normal liquid level of 4 feet, it contained 48,000 gallons (40,000 Imp. gallons). Thus the design criterion of 16,000 gallons per foot of rotor was satisfied.

The plant was inspected on June 16, 1964. The retention time was approximately one day, showing that the experimental conditions were still the same since the end of February. The concentration of MLSS was 6,050 mg/l. A settling test showed 95 per cent settleable solids after thirty minutes, but with a clear supernatant liquid.

Three D.O. surveys were carried out in the course of the day. At a point 6 feet downstream of the rotor in the centre of the ditch near the surface of the liquid, the dissolved oxygen remained virtually constant at 1.5 mg/l. At the downstream bend the D.O. decreased from 0.3 mg/l in the morning to zero in the afternoon. Six feet upstream of the rotor, near the liquid surface, the D.O. remained at zero mg/l throughout the day. The clarified effluent contained 0.0 mg/l D.O. prior to discharge over the weir.

Local conditions permitted the measurement of dissolved oxygen distribution across the ditch and along the depth. The cross-section of the ditch immediately downstream of the rotor was selected. The dissolved oxygen was non-uniformly distributed as shown by the following results: while the D.O. in the centre of the ditch near the surface remained at 1.5 mg/l, the average D.O. near the walls was 0.3 mg/l, and near the bottom in the centre of the ditch a D.O. of 0.2 mg/l was observed. These data indicated that a cross-section of 10 ft. by 4 ft. could not be aerated to a uniform dissolved oxygen concentration by a 3-ft. long cage rotor.

On the day of the plant inspection the liquid level in the ditch was 49 inches, corresponding to a rotor immersion of 7 inches. The speed of rotation of the rotor was 80 rpm, giving an oxygenation capacity of 2.5 lb. O_2 per hr. per ft. of rotor, or 2.5 x 3 x 24 = 180 lb. O_2 per day. The average BOD loading was 80 lb. per day. Therefore the OC/load ratio was $\frac{180}{80}$ = 2.25 lb. O_2 per lb. BOD applied.

The available rate of aeration in mixed liquor can be determined when the effects of mixed liquor on oxygen transfer are allowed for. Due to the high MLSS concentration, a 10 per cent decrease is assumed for the oxygen saturation value as compared with tapwater. The transfer coefficient in mixed liquor is assumed to be 85 per cent of its value in tapwater. The available rate of aeration then becomes

$$\frac{2.5 \times 3 \times 0.85 \times 0.90 \times 10^6}{49{,}000 \times 8.34} = 14 \text{ mg/l/hr. at zero mg/l D.O.}$$

The rate of oxygen utilization of the mixed liquor was also determined. From three tests an average rate of 19.7 mg/l/hr. was calculated. Thus the rate of oxygen utilization was significantly greater than the calculated rate of aeration. Although a dissolved oxygen concentration of 1.5 mg/l was observed downstream of the rotor, the average D.O. in the ditch was probably less than 0.5 mg/l. In the presence of such low D.O. concentrations the quality of the mixed liquor will deteriorate, resulting in an increased rate of oxygen utilization and a further decrease of the residual D.O.

On the basis of the design criteria of the oxidation ditch, viz. OC/load ratio, BOD loading and retention time, the pilot plant should operate satisfactorily under the current experimental conditions. The aeration characteristics of the rotor might improve when the MLSS are reduced to 4,000 mg/l or less. When the residual D.O. increases, the rate of oxygen utilization should decrease to a value more in line with the available rate of aeration.

3.5.3 Capital Cost and Maintenance

Because the oxidation ditch was installed as a research project, information on the capital cost was not obtained,

since a full-scale oxidation ditch could not be constructed in as economical a way as the pilot plant.

Operator time, as required for operation of the plant and chemical analysis of daily samples over the past 15 months, would not be applicable to a full-scale installation.

The only mechanical failures throughout the period of operation of the plant were experienced on the rotor bearings on the drive side. The bearings were replaced every six months.

During the winter operation an ice hood extended from the supporting structure on both the upstream and downstream sides. When warmer weather permitted the ice hood to disappear, a plywood cover was built in the place of the natural ice hood. Although some ice formed on the ditch, extending from the exposed tank wall, never was the ditch fully ice covered. However, the tank walls, rising about 6 feet above the liquid surface, may have had a sheltering effect.

3.6 SUMMARY OF FIELD OBSERVATIONS

1) The operation of several of the full-scale plants left something to be desired due to the lack of experience of the operators with the oxidation ditch sewage treatment process. In some instances inadequate instruction in the routine operating methods and the running of control tests was evident. The correct sampling technique and method of analysis of dissolved oxygen were emphasized, especially in two cases where erroneous D.O. results had led to poor operating conditions.

2) As a result of preliminary observations at each plant changes were recommended and effected in the operation of the plants. For that reason the recorded observations were not necessarily an indication of average operating conditions. Due to the nature of the inspection tour, a follow-up investigation was not possible. Only at Beaverton, Oregon could the plant be visited again a week later to determine the effects of sludge wasting. However, no significant improvement was observed.

3) That the oxidation ditch was capable of BOD removal efficiencies of 90 per cent and greater was confirmed by the operating records of the Beaverton and Regina plants. Both plants were equipped with a clarifier, but the intermittent oxidation ditch cannot be rejected solely on the basis of the unsatisfactory results at Williams Lake.

4) In all cases an activated sludge had accumulated, although the quality and quantity varied over a wide range. The quality of the mixed liquor usually indicated the overall condition of the treatment process. At Beaverton and Regina, the mixed liquor contained suspended solids in the range of 6,000 mg/l, and sometimes higher. The settleability was rather poor, resulting in sludge volume index values of 150 ml per gr. per 30 minutes.

5) Dissolved oxygen surveys showed generally lower D.O.'s than in a conventional activated sludge system. Maximum D.O.'s of up to 1.5 mg/l were found a short distance (i.e. from 6 to 20 ft.) downstream of the rotor. Minimum values down to 0.0 mg/l were observed upstream of the rotor. The clarified effluent contained low D.O. concentrations near 0.0 mg/l, i.e. before being discharged from the clarifier.

6) Special D.O. surveys at Stayton and Regina showed that dissolved oxygen was not distributed uniformly across the width of the ditch, 15 ft. and 6 ft. respectively downstream of the rotors. The maximum D.O. occurred in the centre of the ditch. At Regina it was also found, that at the centre of the ditch the D.O. near the bottom was significantly less than the D.O. near the surface.

7) Winter operation proved no problem at Beaverton and Stayton because of the mild climate in the area. No special precautions were necessary to facilitate operation at Williams Lake in spite of rather severe winter weather. At Regina a plywood cover constructed over the rotor ensured trouble free operation throughout the winter.

8) Mechanical maintenance varied from nil at some plants to quite extensive in one case. The extent of required maintenance could generally be related to equipment design and selection of component parts.

4. ECONOMICS

The capital costs of existing installations are summarized in Table VI. The amounts represent the total cost of the plant excluding the land area occupied by the plant.

Table VI

Capital Cost of Installation

Location	Population	Cost	Funds
Beaverton	1,900*	$ 66,000	U.S.
Stayton	3,500	94,000	U.S.
Williams Lake	2,000	30,000	Can.
Montrose	1,500	31,000	Can.

* Note - in the absence of design data, the equivalent population was estimated.

For an accurate evaluation of the recorded cost data they should be compared with the cost figures of lagoon systems under similar conditions. Cost studies in Oregon indicated that the capital cost of oxidation ditch plants was of the same order of magnitude or marginally greater than the cost of lagoons. Similar studies in British Columbia showed, that in each of three cases, the oxidation ditch was more expensive than a lagoon. In both studies an oxidation ditch plant was taken to consist of a concrete lined ditch and a clarifier.

The tabulated figures compare rather favourably with average costs of lagoon systems in Ontario: $63,000 for a population of 2,000, and $122,000 for a population of 5,000. The cost of the required land area is not included. To establish the economic feasibility of the oxidation ditch it is not sufficient to take note of the actual costs of existing installations in relation to the respective design contributory populations.

A recent survey of sewage treatment facilities in Sweden (10) showed a total of 79 oxidation ditch plants in operation, all except one serving populations smaller than 1,000. Most plants were in continuous operation and were therefore equipped with clarifiers. A cost comparison of plants for a range of populations from 100 to 2,000 people showed the oxidation ditch to be more expensive than the lagoon.

Because of the variation of land costs and the marked reduction of land requirements of an oxidation ditch compared to a lagoon, the oxidation ditch may prove to be more economical in specific locations in Ontario.

The annual operating costs of an oxidation ditch plant will be considerably higher than for a lagoon. Power is required for the comminutor, rotor and return sludge pump. Mechanical maintenance costs must be included, as well as daily operator time of one man-hour. Engineering studies in British Columbia estimated the annual operating and maintenance costs to range from $3,000 to $4,500 for design populations ranging from 3,500 to 6,000 persons. Actual average annual operating and maintenance costs were not obtained for the inspected installations.

5. CONCLUSIONS

1) On the basis of field observations the oxidation ditch provides adequate treatment under suitable operating conditions. Effluent quality is similar to that achieved with the conventional activated sludge process, provided that the mixed liquor suspended solids content can be maintained at a constant level of about 4,000 mg/l by regular wasting of excess sludge. If the mixed liquor solids are allowed to build up until the system reaches a balance, solids will be wasted in the plant effluent, increasing its suspended solids and BOD content. Therefore, the BOD and suspended solids removal will be less, approaching that of an extended aeration system.

2) An oxidation ditch treatment plant should consist of: an oxidation ditch, a clarifier with return sludge pump and skimming device, sludge wasting facilities, plus any other equipment which may be considered necessary.

3) Design criteria follow, which are mainly applicable to the treatment of domestic sewage (2,5).

Ditch volume can be calculated from the BOD loading, which is not to exceed 13 lb. BOD per day per 1,000 cu. ft. of ditch volume, and from the retention time of 24 hr. at average design flow.

Required rotor length is calculated from the oxygen requirements of the system of 2 lb. O_2 per lb. BOD applied. The test data obtained at the Iowa State University (6) should be used. To provide adequate circulation, the ditch volume per foot of rotor should not exceed 16,000 gallons (13,300 Imp. gallons) for a concrete lined ditch. More rotor length is required for an unlined ditch, since the ditch volume per foot of rotor is reduced to 13,000 gallons (10,800 Imp. gallons).

Liquid depth in the ditch may vary from 3 to 5 feet.

The clarifier should provide a retention time of 3 hr. and a return rate of 100 per cent at average design flow.

When the <u>sludge wasting facilities</u> consist of drying beds, an area of 1 sq. ft. must be provided per population equivalent.

4) Although the oxidation ditch was originally designed as an unlined, dug ditch, and although a good number of this type of installation in Europe has given satisfactory results, experience with unlined ditches in British Columbia has generally been unsatisfactory. Intermittent operation proved also unsatisfactory. Therefore, only a concrete-lined oxidation ditch equipped with a clarifier should be accepted.

5) Available literature and operating experience have not indicated an upper limit for the design population, for which an oxidation ditch would still be preferable. Most plants in Europe were built for populations smaller than 3,000. The existing installations in Canada and the United States serve populations ranging from 1,000 to 3,500, while several plants are being designed for populations of 5,000 and 6,000. Perhaps a design population of 10,000 is a reasonable maximum, because of the large area required for a plant of this size.

6) Although the average operator time required for operation of the inspected installations is approximately one man-hour daily, it is realized that the actual operator time will depend on the maintenance requirements of a particular installation. But in any event, an oxidation ditch plant can be operated on a part-time basis.

7) Operation under winter conditions should not give serious problems in Ontario, particularly when the ditch is in continuous operation. It would appear an added disadvantage to suspend the rotor from an overhead structure, since this facilitates an ice build-up on that structure. If necessary a protective cover can be erected over the rotor for the duration of the cold weather. Experience in Europe has shown that purification of the wastewater continued even when the ditch was fully ice-covered, as long as the rotor remained in operation.

8) As a result of the fluctuation of dissolved oxygen in the ditch between concentrations of 1.5 and near zero mg/l, the clarified effluent is usually very low in dissolved oxygen. When the BOD is low also, the lack of oxygen will not be a drawback provided that the effluent can be aerated, before it reaches the receiving stream. There is some recent evidence (2), that a dual process occurs in the oxidation ditch, viz. nitrification followed by denitrification, due to the regular fluctuation of the dissolved oxygen. Thus an effluent could be produced that is low in BOD as well as ammonia and nitrates.

9) Operating experience has proven it desirable that the immersion of the rotor can be adjusted. This could be accomplished by installing an adjustable overflow weir on the ditch, or on the clarifier when the liquid level in the ditch is controlled by the liquid level in the clarifier. As suggested under No. 7 in the foregoing, the rotor should preferably not be mounted on an overhead support, to prevent problems with winter operation.

10) Since an adequately designed and properly operated oxidation ditch is entirely odour-free, in spite of occasionally low dissolved oxygen concentrations in certain sections of the ditch, the plant may be located in close proximity to residential or commercial areas. This would result in a considerable reduction of the required length of sewer lines conveying the raw sewage to the plant.

11) Where land values are high, or where flat land is scarce, the oxidation ditch should be considered more suitable than a lagoon, because of the much smaller area required.

6. RECOMMENDATIONS

1) It is recommended, that the oxidation ditch be accepted as a means of secondary treatment of wastewater of small municipalities, suitable for installation in Ontario. The oxidation ditch offers an alternate choice to treatment by conventional or aerated lagoons. A preliminary study by the consulting engineer should indicate which treatment process is economically feasible for a particular location.

2) The first oxidation ditch installed in Ontario should be subjected to a thorough investigation to confirm the European experience. Special attention should be given to the oxygenation capacity of the rotor under process conditions, and to the reduction of free ammonia and the production of nitrites and nitrates.

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